SEISMIC RESPONSE OF STEEL BUILDING FRAME WITH WEAK BEAM-TO-STRONG COLUMN BY SUBSTRUCTURE PSEUDO DYNAMIC TESTING METHOD

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SUMMARY

In the last decade, the substructure method has been introduced into the pseudo dynamic testing method. This method is effective to evaluate such as the performance of seismic energy absorbing devices, because it is clear to select a tested part of the object. In many cases, damages are distributed to not only some limited zone of the structure but also many structural elements. In these cases, how can be applied this testing method? In this paper, some of the issues for its application are solved.

For this purpose, the test structure is chosen as the six-story steel moment-resisting frame building with one span, which would be expected to collapse with plastic hinges at beam ends. Damages are mainly concentrated to the lower stories, whereas the upper stories yield also slightly under severe earthquakes. The tested part is the lower three stories of the structure, which is a 2/3 scaled-model of the real size. Two tests are continuously implemented using different levels of the scaled wave of Kobe Earthquake 1995.

The boundary condition at the top of the specimen is verified to give sufficient results by the elastic loading test and the simulation analysis. The computed part of the structure is kept in elasticity during the response loading test, whereas it would be expected to yield a little according to the pre-analysis. But, at the same time, the analytical results show that the assumption in the computation of the upper three stories would be reasonable in the case of this structure. The elastic-plastic response of the test structure against severe earthquakes is verified by the substructure pseudo dynamic testing method.

INTRODUCTION

The pseudo dynamic testing method has been developed and was verified to be effective as the testing method which can give the seismic response of building structures [Okamoto et al., 1986], [Nakashima and Kato, 1987]. In the test, the entire part of the structure must be constructed as the test specimen, then it takes a large amount of cost and large-scale facilities are needed. Dr. Nakashima et al. introduced the substructure method into the computer-on-line-structural tests which evaluate the seismic response of buildings, i.e. substructure pseudo dynamic testing method, and developed the algorithm for this testing method [Nakashima et al., 1988 and 1989]. The seismic response is obtained by the simultaneous execution of the structural testing and computer analysis; the tested part of the structure is the part which would be expected to have an elastic-plastic behavior, and the computed part is the rest part which will behave in elasticity. The implicit integration is applied to the computed part of the structure, and the tested part is solved by the explicit integration technique.

This method is effective to evaluate such as the performance of the seismic energy absorbing devices, because it is clear to select the tested part of the building structure. In many cases, the damages are distributed to not only

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some limited zone of the structure but also many structural elements. In these cases, how can be applied this testing method? This paper solves some of the issues for its application by the scaled model test of the steel moment resisting frame.

SUBSTRUCTURE PSEUDO DYNAMIC TESTING METHOD EXAMINED

Substructure Model

The pseudo dynamic test is only one variation of the numerical analyses using experimental information on the restoring force characteristics of the structure. In this meaning, the pseudo dynamic test does not need the test specimen representing the whole structure. Only a part of the structure, whose restoring force characteristics are difficult to model, may be fabricated and tested; the rest can be handled within the computer. At that point, the loading part of the analyzed structure should be carefully selected so that the experiment on only this part makes sense.

As the first step of the test, the elastic stiffness matrix for the entire structure [**K***], that for the tested part [$\mathbf{K}_{ts}$], and that for the computed one of the structure [$\mathbf{K}_{an}$] are constructed as follows. When the story number is 6 and two axial forces are controlled as shown in Fig. 1 because of one bay in lateral loading direction, the number of variables are 8, i.e. 6 variables for lateral displacement and 2 variables for vertical forces at the top of the tested part. Therefore, the matrix size is 8x8. [$\mathbf{K}_{ts}$] is built up from beams and columns in the tested part, and [**K***] is from all the members of the structure.

$$[\mathbf{K}_{an}] = [\mathbf{K}^*] - [\mathbf{K}_{ts}] \tag{1}$$

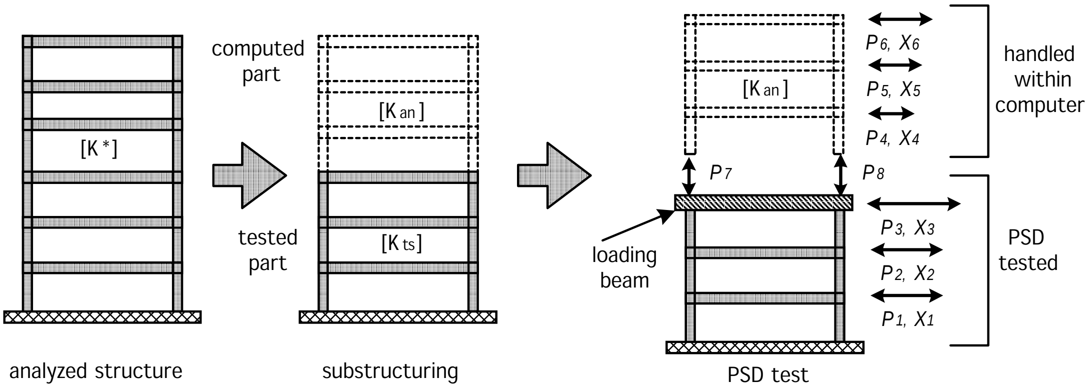


Fig. 1 Substructure Pseudo Dynamic Testing Method Applied to Building Frame

Integration Process

The state of the structure is described by Equation 2.

$$[\mathbf{M}]\{X'' + y''\} + [\mathbf{C}]\{X'\} + \{F\} = \{0\} \tag{2}$$

where [**M**], [**C**] : mass matrix and damping matrix of the structure, respectively
{X''}, {X'}, {X} : acceleration vector, velocity vector and displacement vector, respectively
{F} : resisting force vector, equal to [**K***]{X} in elasticity
{y''} : ground acceleration

$$\{\underline{\boldsymbol{X}}_{n+1}\} = \{X_n\} + \Delta t\{X_n'\} + \Delta t^2\{X_n''\}/4 \tag{3}$$

$$\{X_{n+1}\} = \{\underline{\boldsymbol{X}}_{n+1}\} + \Delta t^2\{X_{n+1}''\}/4 \tag{4}$$

$$\{X_{n+1}'\} = \{X_n'\} + \Delta t\{X_n''\}/2 + \Delta t\{X_{n+1}''\}/2 \tag{5}$$

$$[\mathbf{M}]\{X_{n+1}'' + y_{n+1}''\} + [\mathbf{C}]\{X_{n+1}'\} + \{F_{n+1}\} = \{0\} \tag{6}$$

$$\{F_{n+1}\} = \{P_{n+1}\} + [\mathbf{K^*}]\{\{X_{n+1}\} - \{\underline{\boldsymbol{X}}_{n+1}\}\} = \{P_{n+1}\} + \Delta t^2\,[\mathbf{K^*}]\{X_{n+1}''\}/4 \tag{7}$$

$$\{X_{n+1}''\} = -\,[[\mathbf{M}] + \Delta t[\mathbf{C}]/2 + \Delta t^2\,[\mathbf{K^*}]/4]^{-1}\{[\mathbf{M}]\,\{y_{n+1}''\} + [\mathbf{C}]\{X_n'\} + \Delta t[\mathbf{C}]\,\{X_n''\}/2 + \{P_{n+1}\}\} \tag{8}$$

At the calculation step of the time t_n, the variables are known as $\{X_n\}$, $\{X_n'\}$ and $\{X_n''\}$ of the displacement, velocity and acceleration, respectively. At the time $t_{n+1} = t_n+\Delta t$, the predictor vector of displacement is defined by Equation 3. The displacement of the predictor $\{\underline{\boldsymbol{X}}_{n+1}\}$ is applied to the test structure. The lateral reaction forces of the tested stories are measurement by the load cells mounted to the test specimen. The results include the inelastic behavior of the specimen. The lateral restoring forces (P_4, P_5, P_6) of the computed stories and the vertical forces (P_7, P_8) are obtained by $[\mathbf{K}_{an}]\{\underline{\boldsymbol{X}}_{n+1}\}$. Then, these lateral restoring forces and two vertical ones are presented by $\{P_{n+1}\}$. The corrector vector has a relation to the predictor as described in Equation 4. The velocity and restoring force of the structure, are the functions of the corrector and predictor as described in Equations 5 and 7 respectively. The $\{X_{n+1}''\}$ vector in these three equations are given by Equation 8. Equation 7 means that the inelastic behavior of the structure is presented by the first term $\{P_{n+1}\}$ of the right hand side. And the second term of the equation shows that the restoration response in the duration between the corrector and predictor of displacement is obtained under the assumption of elastic stiffness. As a result, the acceleration response $\{X_{n+1}''\}$ at the time t_{n+1} is calculated from the restoring forces of the test structure $\{P_{n+1}\}$ and the values about previous step $t=t_n$ as mentioned in Equation 8.

Table 1 Size of Members

member	Story or floor	prototype	scaled model
column	1	box-400x400x22	box-300x300x19
	2	box-400x400x19	box-300x300x16
	3	box-400x400x19	box-300x300x12
	4, 5, 6	box-400x400x16	box-300x300x12
beam	2, 3, 4	H-588x300x12x20	H-500x200x10x16
	5	H-488x300x11x18	H-446x199x8x12
	6, Rf	H-482x300x11x15	H-446x199x8x12

unit: mm

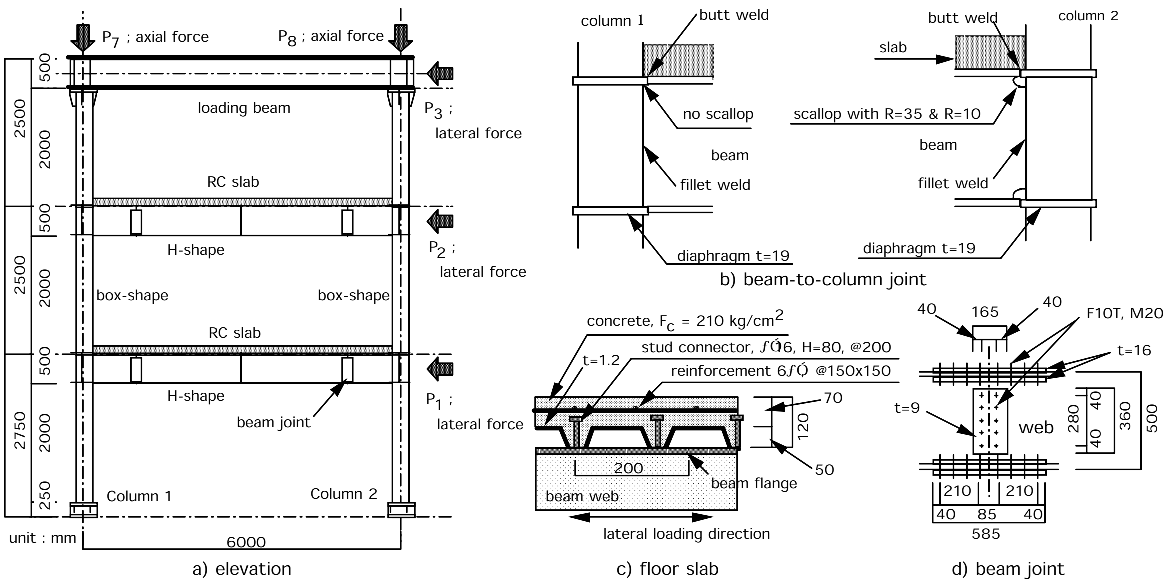


Fig. 2 Test Specimen

PROTOTYPE STRUCTURE, SCALED MODEL AND TEST SPECIMEN

The prototype structure is chosen as a six-story moment-resisting steel frame with a 9.0m length bay in the both horizontal directions and 3.75m height in each story, which would expected to collapse with the plastic hinges at beam ends. Because of the capacity limitation of the testing facility, the scaled model is two thirds of the prototype structure. Members of the two structures are listed in Table 1. According to pre-analyses, damages are mainly concentrated to the lower three stories of these structures. Therefore, these stories are designed as a test specimen.

TEST SETUP

At the top of the specimen, a rigid loading beam is fastened by high strength bolts in order to transfer lateral forces from actuators to the columns of the specimen. By actuators with 3MN(300 tf) capacity, vertical forces are applied at the top of each column through the loading beam. Two lateral actuators with 1MN(100tf) capacity are set at the loading beam to prevent the out of plane deformation and one actuator with the same capacity is mounted at the second and third floors of the specimen. In the loading, vertical actuators are controlled by force and kept vertical position with a lateral slide system, which is installed between actuators and a rigid reaction frame.

Table 2 Models of Simulation and Response Frame Analysis

case	simulation or response frame analysis	moment inertia of 4th floor beam : I_4
1	simulation	$I_4 = I_{act}$=3.30 I_{beam}
2	ditto	I_4 = 2.0 I_{act}= 6.60 I_{beam}
3	ditto	I_4 = 0.5 I_{act}= 1.66 I_{beam}
4	ditto	$I_4 = I_{beam}$= 1.35 I_s
5	response frame analysis	$I_4 = I_{beam}$ (same as above)

cf I_{act} : moment inertia of section of loading beam mounted to the test specimen
I_{beam} : moment inertia of composite beam section
I_s : moment inertia of steel beam section
1.35 : composite effect of concrete floor slab section

Table 3 Comparison of Response between Simulation and Response Frame Analysis

Response ratio		Case-1	Case-2	Case-3	Case-4
1st floor displacement	Max.	1.030	1.017	1.038	1.043
	Min.	1.053	1.023	1.067	1.095
2nd floor displacement	Max.	1.026	1.013	1.029	1.034
	Min.	1.047	1.024	1.057	1.086
3rd floor displacement	Max.	0.987	0.995	0.977	0.978
	Min.	1.006	1.005	1.002	1.024
Top floor displacement	Max.	1.001	1.001	0.996	0.966
	Min.	1.016	1.007	1.016	1.001
End rotation of 4th floor column	Max.	0.951	0.978	0.930	0.494
	Min.	0.963	0.985	0.948	0.509

Values are the response of each simulation case divided by the response of Case-5.

EFFECT OF BOUNDARY CONDITION AT LOADING BEAM

At present, the loading system can not control rotation of the nodes at the forth floor of the test structure. Therefore, only lateral displacements of the floors and vertical forces at the top of the test specimen are controlled in the loading. Moreover the loading beam is 3.3 times stiffer than the composite beam of the scaled model structure. This is because that the lateral forces can be distributed to columns. In order to investigate the effect of these different boundary conditions from the model frame on its overall seismic response, elastic responses are simulated on the substructure pseudo dynamic testing system. The analytical parameter in the simulation is the stiffness of the loading beam (see Table 2). Here, Case-1 is an actually loaded one. The response results are compared with those of the response frame analysis named Case-5 in Table 3. In the table, displacements or end rotations of the simulated cases are divided by those responses of Case-5. Among four simulation cases, Case-2 gives the best results fitted to the response of Case-5. As Case-1 is a simulation of the experiment, it has a better and sufficient correspondence with that of the response frame analysis (Case-5) of the entire structure.

EFFECT OF YIELDING IN UPPER STORIES

The upper stories of the scaled model are also damaged under severe earthquakes like the Kobe Earthquake of Jan. 17 1995, whereas it was designed to have a major damage in the lower three stories. In the substructure pseudo dynamic testing program in Building Research Institute, the computed part of the model is dealt with being in elasticity. Before testing, we estimated the effect of the assumption about the computed part in the loading on the overall response of the model structure. Then, the elastic-plastic response of the three models listed in Table 4 are analyzed using the DRAIN2DX response program. The reduced stiffness in the case-C presents the equivalent elastic stiffness considering to the plastic response of the prototype structure. The displacement response of Case-C after 5 seconds shows the different frequency from the Case-A. This means that it is very difficult to adjust the equivalent stiffness for upper three stories. At some peak responses, a little difference exists between Case-A and Case-B. But as a whole, the response against the Kobe Earthquake NS component (measured by Japan Meteorological Agency) shown in Fig. 3 can lead to the conclusion that the sufficient results would be obtained even if the upper three stories could be dealt with being elastic as-is configuration in the test.

Table 4 Models of Response Frame Analyses

case	model of 1st to 3rd story	model of 4th to 6th story
Case-A	elastic-plastic	elastic-plastic
Case-B	elastic-plastic	elastic, stiffness is equal to that of Case-A
Case-C	elastic-plastic	elastic, stiffness is 2/3 of Case-A

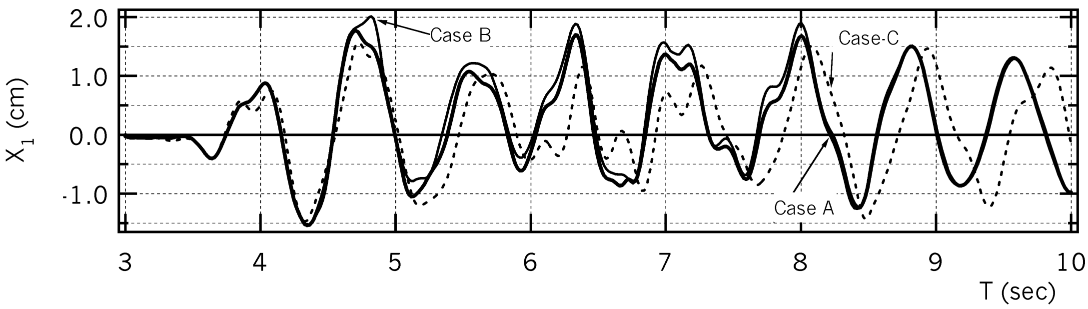


Fig. 3 Displacement Response of First Story in Case-A, Case-B and Case-C

RESULTS OF SUBSTRUCTURE PSEUDO DYNAMIC TEST AND ANALYSIS

Two input tests were continuously implemented. The first one is an elastic response test against the JMA Kobe Earthquake NS component scaled by 1/9, and the second is an elastic-plastic response to the same wave, but scaled by 5/9, of which intensity is equivalent to the velocity of 50 cm/sec. The first 4.12 seconds of the original record was omitted because of very small amplitude. The time axis is reduced to 0.817 in consideration of the similarity law. The damping ratio is assumed to be 2% of the critical damping and proportional to the elastic stiffness. The inter-story displacement of the second floor is drawn in the upper figure of Fig. 4 in comparison with the analytical results by the DRAIN2DX program. The column bases are fasted to the testing floor by high strength bolts. As shown in Fig. 6, lateral slide and end rotation occur at the foot of the first story columns. Because the friction force is not sufficient between the base plate and testing floor, and the base plate is bent and rotated due to the bending moment. Therefore, the model with elastic slide and elastic end rotation is considered. The elastic stiffness of these springs is obtained from the test results (dotted lines drawn in Fig.6). Also, in the analysis, the yield strength of the steel materials and the compressive strength of the concrete are the results of the material tests. Of course, there is no yielding in the members against this level of input. The elastic responses have a good coincidence in the first 1.4 seconds. But after that, the peak values are different and the results of the analysis are bigger than the test. As seen the Q-X relation of Fig.5, the inclination of the analytical results is steeper than that of the test. The test specimen has a smaller response in comparison with the analysis after 1.4 seconds. This might be a damping effect of connections at the column foot, as shown in Fig. 6.

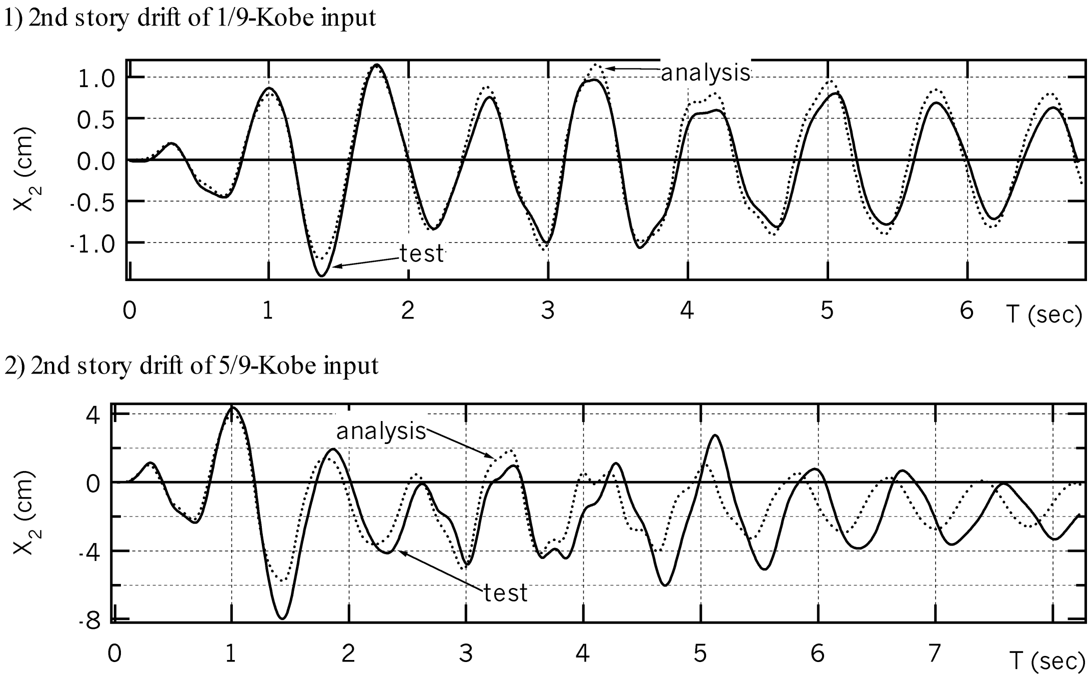


Fig. 4 Responses of Second Story Drift of Two Tests

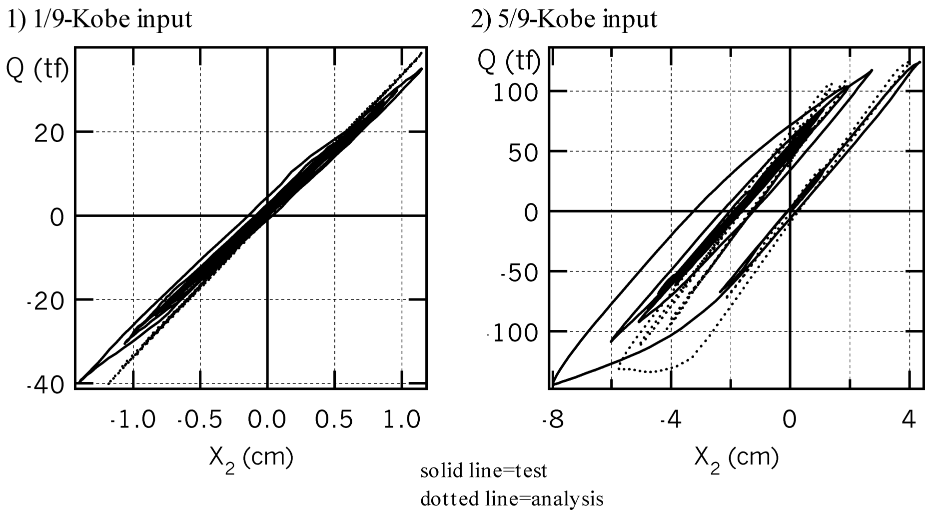
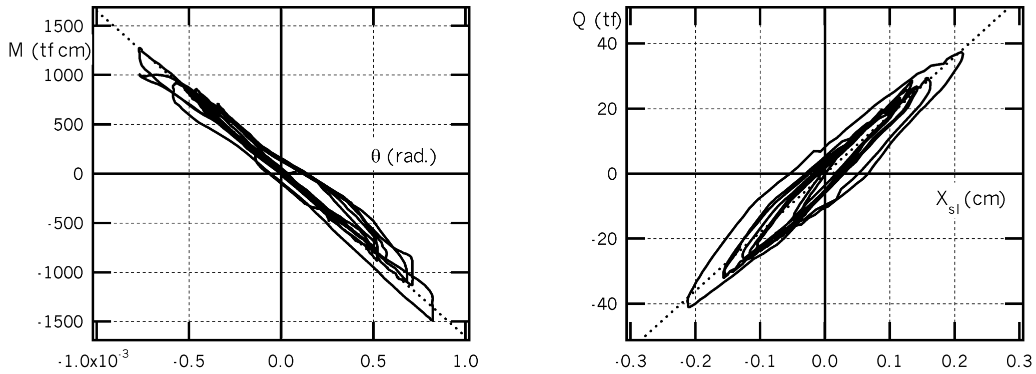


Fig. 5 Story Shear Force vs. Story Drift of Second Story Response

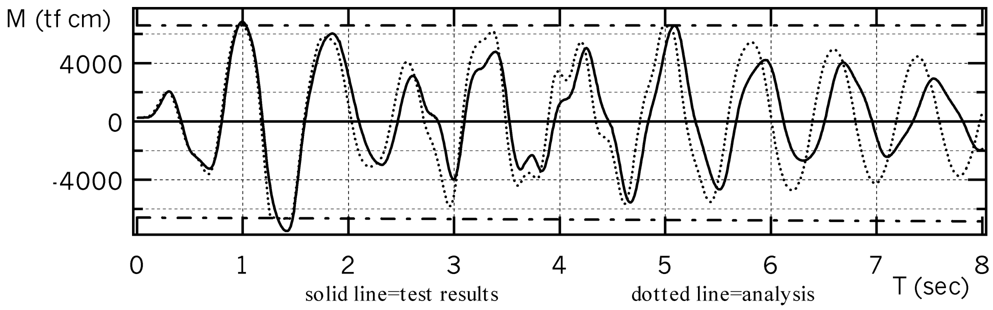
In the elastic-plastic response, overall shapes of the curves of the time history are similar, especially the first 1.3 seconds show a prefect comparison between the analysis and the experiment (see Fig. 4-2)). But after the peak at 1.4 seconds, some difference between them appears in the time history. This is more evident to see the Q-X relation of the second story in Fig.5. Fig. 7 shows the end-moment of the second floor beam of the test in comparison with the analytical results. These moments in members are calculated from the output of strain gauges glued on the steel surface at the section, which would remain in elasticity. In the figure, the full plastic moment of the steel beam is marked. At about 1.2 seconds, the end-moment reaches the full plastic moment and the beam ends yield. After that, the analysis can not follow the test results exactly. The main reason seems that the analysis dose not follow the expansion of yielding in the structural elements corresponding to the increase of stress and the stiffness reduction due to the repeated yielding of steel, so called Bauschinger's effect.



dotted line : K_r=1.65x10^6 tf cm/rad.
1) Moment-end rotation relation of lower end of column-2

dotted line : K_{sl}=1800 tf/cm
2) Story shear force-to-lateral slide at basement

Fig. 6 Lateral Slide and End-Rotation at First Story Column Base under 1/9-Kobe Test



solid line=test results dotted line=analysis
dotted & dashed line=full plastic moment of steel beam

Fig. 7 Comparison of Face Moment of Second Floor Beam between Test, Analysis and Mp

CONCLUSION

The substructure pseudo dynamic testing method was applied to the steel frame with weak-beam and strong-column system in order to get the seismic response of the whole structure to a severe earthquake.
1) The tested part of the structure, which is the lower three stories of the six-story, was loaded by horizontal actuators and vertical ones. The boundary condition at the top of the specimen was examined by the pre-analyses and verified by the elastic loading test to give sufficient results.
2) The computed part of the structure, which is the upper three stories, was kept in elasticity during the response loading test, whereas they would be expected to yield a little according to the pre-analysis using the DRAIN2DX program. But, at the same time, the analytical results showed that the assumption in the computation of the upper three stories would be reasonable in the case of this structure.
3) The elastic-plastic response of the test structure was obtained which can be followed by the analysis of the DRAIN2DX program as a whole. But strictly speaking, more sophisticated analyses would be needed in order to explain the post yielding response, which can deal with extension of the yielding zone of members and more realistic stress-strain relation of steel materials.

ACKNOWLEDGMENT

The authors wish to express the deepest appreciation to Mr. Sasaki N. of Mitsubishi Heavy Industries LTD. and Dr. Yamamoto T. of Ryousen Engineers LTD. for their participation to develop and simulate the substructure pseudo dynamic testing method on steel building frames.

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